

Memorandum


*Serious drought,
Help save water!*


To: MR. RON TSUNG
District Office Chief
Design East – Contra Costa


Date: January 15, 2015


Attention: Qi Fu

File: 04-ALA-580 PM 5.0
04-1SS030
EFIS# 0400020869-0
North Flynn Road
Soldier Beam Wall with
Ground Tieback Anchors
Storm Damage

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Subject: **REVISED FOUNDATION REPORT**

I. INTRODUCTION

This Foundation Report (FR) supersedes our June 2, 2014 FR recommendations for the repair of the roadway distress and slope failure located on westbound of the State Route 580 PM 5, close to Livermore, Alameda County (see Figure 1).

- We have performed a geotechnical investigation to determine the possible causes of the slope failures, and developed a repair plan. The scope of work includes the following:
- Several site visits for reconnaissance of the distressed roadway and slopes.
- Background reviews.
- Subsurface exploration consisting of four exploratory borings advanced to approximate depths of 50 to 75 ft on July 2012.
- Two exploratory boring were converted to a slope inclinometer and a piezometer. Periodic readings of the slope inclinometers/piezometers were recorded.
- Engineering analyses and preparation of this repair recommendations.

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II. BACKGROUND

During the Winter of 1998-1999, severe storms saturated and weakened the slopes at this location. An emergency repair was implemented by a Director's Order project. The scope of the project was for a medium-life solution consisting of geogrid reinforced embankment and was built during the summer of 1999. This repair exhibited moderate success up until cracking was observed in the number 1 lane in October 2008. The overall combined length of these cracks is approximately 1200-feet. It appears that there has been constant creeping of the left shoulder away from I-580 towards the southeast resulting in continuous maintenance repairs of the number 1 and 2 lanes.

III. PHYSICAL SETTINGS

III.1 Climate

The climate in the project area is characterized as Mediterranean, with warm, dry summers and cool, moist winters. The average annual temperature varies from 56° F and 62° F with the mean maximum temperature of 89° F, occurring in July and the mean low temperature occurring in January of 35° F. The maximum temperature reported in Livermore area was 115° F and the lowest reported temperature was 19° F. On the average, freezing temperatures occur 7 to 10 days each year; however, freeze-thaw conditions have a low potential to impact the proposed project.

The average annual precipitation for the Livermore area over 69 years is 15 inches, with most of the precipitation falling between the months of November and March. Winter storms that move through the area are usually of moderate duration and intensity, but sometimes the rainfall is heavy enough to cause flooding.

III.2 Topography and Drainage

The project site is located in Alameda County approximately 3.5 miles east of the Livermore Valley and 4.0 miles southwest of the San Joaquin Valley. The stretch of highway is typically referred to as Altamont Pass. The area that Highway 580 is located in is approximately 900 feet in elevation and is adjacent to Mountain House Creek. Drainage within the project area is characterized as uncontrolled sheet flow into the Altamont and Mountain House rivers, and then into the San Joaquin River, which drains into the Suisun Bay

IV. GEOLOGY

IV.1 Regional Geologic Overview

The project is located in the Coast Range Geomorphic Province of Central California, a series of northwest-trending mountain ranges (2,000 to 4,000, occasionally 6,000 feet elevation above sea level), and intermountain valleys, bounded in the east by the Great Valley and to the west by the Pacific Ocean. The Coast Ranges are composed of thick Cenozoic sedimentary and volcanic strata overlying Mesozoic metamorphic basement rock. The northern and southern ranges are separated by a depression containing the San Francisco Bay. The Coast Ranges are subparallel to the active San Andreas Fault, which is more than 600 miles long, extending from Pt. Arena to the Gulf of California.

IV.2 Soils

The majority of the project is underlain by Altamont clay.¹ This clay is residuum weathered from sandstone and shale. The Altamont clay is classified as Hydrologic Soil Group D. "...Group D - Soils in this group have high runoff potential when thoroughly wet."² The soils within the proposed project limits have limiting features. Such as the potential to shrink-swell, low soil strength and unstable excavations. Table 1: Project Soils, describes the soils and soil features of the soils found at the site (The USDA, NRCS; Custom Soil Resource Report for Alameda County, California; 2012 can be supplied upon request.)

Table 1: Project Soils

Soils	Soil Symbol	Hydro-logic Group	Percent Slope	k Factor	Shrink-Swell	Soil Strength	Stability of Excavation
				The numbers in the value columns range from 0.01 to 1.00. The larger the value, the greater the potential limitation.*			
Altamont clay	AmE2	D	30-45	0.20	1.00	1.00	1.00

*Limitations indicates that the soil has features that are favorable (0.1) to unfavorable (1.0) for the specified use.

Data from USDA, NRCS; Custom Soil Resource Report for Alameda County, California; 2012

¹ <http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx>

² Ibid

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Table 1 presents the shrink-swell, soil strength and stability of excavation factors for project soils. These factors are 1.00, the highest given for the potential construction limitations for the soil characteristics. The native soils at the site have a high potential for shrink-swell, low soil strength and poor excavation stability. However the project as planned is located within mostly embankment fill, therefore there will be minimal amount of native soil.

IV.3 Site Geology

According to the Preliminary Geologic Map Emphasizing Bedrock Formations in Alameda County, California (Graymer and others, 1996), the site is underlain by unnamed shale, siltstone and sandstone (Kcu). This unit is Late Cretaceous in age and is described as medium to coarse grained, light gray, clean sandstone. Grains include quartz, feldspar, and biotite. Spherical weathering is common. In places the clean sandstone is interbedded with fine to medium grained, biotite and muscovite bearing wacke with mudstone rip-up clasts. Sandstone beds form packages up to 10 meters thick with 1 to 2 meters of interbedded siltstone and mudstone. The relevant portion of the map is included as Figure 2, Vicinity Geologic Map.

IV.4 Seismicity

Regional Seismicity and Faults

The San Francisco Bay Area is one of the most active seismic regions in the United States. Three major faults trend northwest through the Bay Area and have generated about 12 earthquakes per century large enough to cause significant structural damage. These earthquakes occur on the San Andreas, Hayward, and Calaveras Faults, which are part of the San Andreas Fault system that extends for at least 700 miles along the California Coast, and includes.

The U. S. Geological Survey concluded that there is a 62 percent probability for at least one "large" earthquake of magnitude 6.7 or greater in the Bay Area before 2032. There could also be more than one earthquake of this magnitude and that numerous "moderate" earthquakes of about magnitude 6 are probable before 2032. The San Andreas Fault is estimated to have a 21 percent probability of producing a magnitude 6.7 or larger earthquake by the Year 2032 (WGCEP, 2003). The probability of the Hayward, Calaveras, and Greenville Faults producing a similar size earthquake during the same time period is 27 percent, 11 percent and 3 percent, respectively (See Figure 3, San Francisco Bay Region Earthquake Probability Map).

Site

According to the latest California Seismic Hazard Map Version 2.0.4 (USGS, 2008), which is based on the United States Geological Survey (USGS) and California Geological Survey (CGS) maps, the nearest active faults are the Greenville Fault and the Las Positas Fault Zone. The site is not located in an Alquist-Priolo Earthquake Fault Zone. The Greenville fault is a

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strike slip fault, has a Maximum Magnitude (Mmax) of 6.9, and is located approximately 2.18 miles to the west of the project. The Las Positas Fault zone is also a strike slip fault, has a Mmax of 6.4 and is located approximately 2.8 miles southwest of the site. The fault distances were measured on Google Earth and represent the horizontal distances from the fault traces or surface projections of the top of rupture planes to the project site.

Site Ground Motions

Since geophysical testing, including shear wave velocity, was not performed at the site, we determined the Vs30 based on the geologic map and logs of test borings. Vs30 refers to the average shear wave velocity in the upper 30 meters of the soil/rock profile and is a measure of the near surface soil stiffness. As noted in section 1.1, Geologic Conditions, the geologic map for the area indicates that the site is underlain by interbedded sandstone, siltstone and shale bedrock. Our logs of test borings indicate that the site is underlain by up to approximately 30 feet of clayey soil, underlain by soft to hard interbedded sandstone and mudstone. Based on the boring logs and geologic map, we assigned a NEHRP class C to the site, which correlates to a shear wave velocity (Vs30) of 560 m/s, soft rock.

We generated Acceleration Response Spectrum (ARS) curves with the Caltrans Deterministic Seismic Hazard Analysis (DSHA) and Probabilistic Seismic Hazard Analysis (PSHA) version 2.0.4 using a 975-year return period (5% probability of exceedance in 50 years). Due to the high seismicity of the site, the PSHA response spectra were higher than the deterministic spectra. The probabilistic and deterministic data generated from the curves are listed in table 1 below.

IV.5 Geologic Hazards

The site may be affected by activity along any of the active faults discussed above. Earthquake induce hazards can be categorized as primary and secondary seismic effects.

Primary seismic effects such as ground rupture or surface deformation resulting from differential movement along a fault trace are not expected to occur on the site.

Secondary seismic effects result from various soil responses to ground acceleration. These effects result from activity of any nearby active faults.

- Liquefaction of Natural Ground – Liquefaction is a process by which soil deposits below the water table temporarily lose strength and behave as a viscous liquid rather than a solid, typically during a moderate to large earthquake. In general, very loose to medium dense, clean fine- to medium-grained sand and very soft to firm, low plasticity silts that are relatively free of clay are most susceptible to liquefaction. Earthquake-induced ground

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shaking can cause these loose or soft materials to densify, resulting in increased pore water pressures and an upward movement of groundwater that may result in a liquefied condition. Depending on the weight of the structure, the depth to the liquefied stratum and the nature of the overlying soils, structures situated above such temporarily liquefied soils may sink or tilt, causing significant structural damage.

According to the State of California Preliminary Seismic Hazard Zones Map for the Altamont Quadrangle (2008), the site is not located in an area where historical occurrence of earthquake-induced liquefaction, or local geological, geotechnical and groundwater conditions indicate potential for permanent earthquake-induced ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required (see Figure 4, State Seismic Hazard Zones Map).

- Cracking – Lurch cracks may develop in the silty and clayey soil overlying the site. The potential for lurch cracking will be higher in the rainy periods when the soil is saturated. The hazard from cracking is considered minimal.
- Differential Compaction – During moderate and large earthquakes, soft or loose, natural or fill soils can become densified and consolidate, often unevenly across a site. The segment of I-580 within the project limits was originally constructed approximately 50 years ago using cut and fill techniques, with fills up to 80 feet in thickness. Given the relatively old age of the fill, a significant amount of consolidation of the fill has likely already occurred, as noted above having occurred after the 1980 Livermore earthquake. In our opinion, there is a moderate to high potential for differential compaction to occur to segments of I-580 underlain by fill, especially where fill is over 20 feet in thickness. The potential for differential compaction to occur in areas underlain by cuts into bedrock is nil.
- Ground Shaking - As noted in the Seismicity section above, moderate to large earthquakes are probable along several active faults in the greater Bay Area. Therefore, strong ground shaking should be expected at some time during the design life of the proposed development. The improvements should be designed in accordance with current earthquake resistant standards.
- Shrink Swell – As noted in our Soils Section, the Altamont Clay is prone to relatively large volume changes upon wetting and drying cycles. The soil expansion and/or contraction can cause foundations to shift and roadways to crack. Suitable base material will be needed. In sloping areas, underdrains may be warranted in order to keep moisture from moving beneath roadways.

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V. FIELD INVESTIGATION

V.1 EXPLORATION

A subsurface investigation was conducted between July 17th and July 26th, 2012. The subsurface investigation consisted of four vertical borings RW-12-001 through RW-12-004. In-situ Standard Penetration Test (SPT) blow counts were recorded at 5-foot intervals to evaluate the density/consistency of the on-site soils. Soil samples were collected from the SPT sampler. When rock was encountered, rock-coring methods were used. Selected soil and rock samples were transported during July 2012 to the Caltrans Geotechnical Laboratory, in Sacramento, for testing. Complete test results are attached to this memo.

V.2 SUBSURFACE CONDITIONS

Boring RW-12-001 was drilled to a depth of 75-ft. Boring RW-12-002 was drilled to a depth of 50-ft and a slope inclinometer (SI-1) was installed. Boring RW-12-003 was drilled to a depth of 50-ft. and a slope inclinometer (SI-2) was installed. Boring RW-12-004 was drilled to a depth of 70-ft, and a slope inclinometer was installed. The slope inclinometers have been modified to allow for piezometric readings.

The soil and rock encountered during the subsurface investigation, as interpreted in the field from boring RW-12-001, consists of a 30-ft. layer of very stiff to stiff clay, which is underlain by a 45-ft. layer of soft Mudstone (Sedimentary Rock).

The soil and rock encountered during the subsurface investigation, as interpreted in the field from boring RW-12-002, consists of a 15-ft. layer of very stiff clay which is underlain by a 35-ft. layer of very soft to moderately hard Mudstone (Sedimentary Rock).

The soil and rock encountered during the subsurface investigation, as interpreted in the field from boring RW-12-003, consists of a 20-ft. layer of very stiff to medium soft clay which is underlain by a 30-ft. layer soft Mudstone (Sedimentary Rock).

The soil and rock encountered during the subsurface investigation, as interpreted in the field from boring RW-12-004, consists of a 28-ft. layer of soft to very stiff clay which is underlain by a 42-ft. layer of moderately hard to soft Mudstone/Sandstone (Sedimentary Rock).

Please review the Log of Test Borings (LOTB's), which are attached to this memo.

V.3 GROUNDWATER

Boring RW-12-001 was advanced on July 17th, 2012, and was left open in order to measure the groundwater. It was checked on July 25th, 2012 at 1:00 am, and the water level was measured at 69.4 feet below grade. Subsequent groundwater readings are shown in Table 3:

Table 3, Initial Groundwater Readings

Vertical Boring	Depth below ground Surface (ft)	Date
RW-12-001	64.3: 69.4	7/18/12: 7/25/12
RW-12-002	39.2	7/25/12
RW-12-003	11.3	7/26/12
RW-12-004	Not measured	

The above records represent the low groundwater condition since they were measured in July. The actual groundwater fluctuates with season and expected to be much higher than the above noted measurements.

V.4 LABORATORY TESTING

Selected soil samples retrieved from the borings have been tested to evaluate properties pertinent to our analyses. The types of Laboratory tests performed included the following:

- Atterberg Limits (AASHTO T 89, AASHTO T 90).
- Moisture Content (AASHTO T 265, ASTM D 2216).
- Corrosion Content California Test Methods (CTM 643, CTM 442, CTM 417).
- Mechanical Analysis (ASTM D 422)
- Unconfined Compressive Strength (CTM 211).
- Laboratory test results are attached to this Foundation Report.

VI. SOIL/ROCK GEOTECHNICAL PARAMETERS

Soil/rock strength parameters of the sliding mass were determined using back-analysis of the landslide. This is necessary because it is impractical to estimate the residual shear strength of the soil/rock at the slip-plane by conventional methods. The size of the sliding mass was estimated using slip-plane location based on the SIs data, measured head scarp location, and other geologic features. Our back analysis show that slide soil material in dry condition has an effective friction angle of 14° and cohesion of 0 psf (along the slip-plane) for SF of 1.0. To

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simulate the faster landslide movement during the wet periods, we increased pore water pressure in the sliding mass to reduce the SF to 0.95. The soil strength determined by back-analysis and assumed pore water pressure parameters are summarized in Table 4.

The back analysis was performed using computer program "SLOPE/W". The graphical outputs (generated by the computer program) are attached.

Table 4. Back Calculated Soil/Rock Strength and Pore Water Pressure

Soil/Rock Type	Unit Weight Psf	Internal Friction Angle ϕ Degrees	Cohesion C psf	Pore Water Pressure Parameter r_u	Safety Factor (SF)
Fill	130	14	0	0	~ 1.00
Fill	130	14	0	0.12	~ 0.95

VII. GEOTECHNICAL RECOMMENDATIONS

The excavation of the entire slide material and replacement with compact soil material is not a practical solution. Reinforced embankment used proved to be not effective over long term. Therefore, a most viable repair strategy for this location is to construct a soldier beam ground anchor (tieback) retaining system. The wall will be approximately 25 feet in height and about 1200 feet in length. The wall is located on the downhill slope and requires placement of backfill. Below are our recommendations for the backfill and the wall structure design requirements:

Due to limited space at the both ends of proposed retaining wall transition to connect the existing metal beam guard rails we proposed to add additional concrete barriers type 736 (Mod) based on the type 736 S/SV barrier on CIDH piles at both ends of the soldier beam ground anchor (tieback) wall. At each end the barriers will be approximately 4 feet in height and 50 feet in length. These barriers will be included in the Structure Plans for clarity in construction.

VII.1 SOLDIER BEAM GROUND ANCHOR (TIEBACK) RECOMMENDATIONS

We performed additional slope stability analyses for our proposed repair system using a specified slip-plane and back calculated soil/rock parameters, and pore water pressure condition.

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The analyses were performed to determine the ground anchor (tieback) loads and resulting SF. Below are summary of soil/rock strength parameters, design ground anchor (tieback) loads and soldier piles parameters.

Table 5. Rock/Soil strength parameter

Geologic Unit	Unit Weight (γ) pcf	Friction angle ($^{\circ}$) Degree	Cohesion (c) psf
Slide Soil	130	14	0
Foundation Rock	130	30	0

Table 6 Ground Anchor (Tieback) Loads

TYPE OF LOADING	SF	Ground Anchor (Tieback) load 1	Ground Anchor (Tieback) load 2
		On Retaining Wall	On Retaining Wall
Static	1.3	20 Kips/ft @ 15° angle	20 Kips/ft @ 15° angle
Seismic	> 1.1	26 Kips/ft @ 15° angle °	26 Kips/ft @ 15° angle

The results of the stability analysis are attached

Table 7: Soldier Pile

Wall Height ft	Pile spacing ft	Pile min diameter ft	Pile minimum embedment ft
25	5.7 or 8	2	20

Table 8 Pile Friction and Tip Compression Capacities

Pile shaft friction per unit surface area of the pile length below the dredge line of the wall	Ultimate Kips/sqft	Allowable Kips/sqft
	1.44	0.72 (SF=2)
Pile tip compression bearing pressure per unit tip area of the CIDH pile	68.0	22 (SF=3)

The proposed soldier piles ground anchor (tieback) retaining wall system can be designed using the earth pressures and criteria outline in Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.7 Figure 5.5.5.7.1-1b.

Add traffic load equivalent to a rectangular pressure diagram equivalent to 2 ft of fill applying from the top of the wall to a depth equal to the wall height. .

Based on the results of the above analyses the wall shall be capable of resisting an additional seismic uniform earth pressure estimated to be equal to 17H (psf) applied 0.6H above the base.

Friction Factor between wall and backfill = $2/3$ of Internal Friction Angle
($\delta = 2/3 \phi$)

Based on the above, we recommend using Design Pullout Load of 20 Kips/ft and a Max Pullout Test Load of 26 kips/ft of the wall.

The above recommended design parameters are based on the assumption that an adequate drainage system consisting of horizontal drains spaced at about 30 ft along and near the base of the wall and with 40 ft length shall be provided to prevent the development of hydrostatic pressure behind the wall. If complete drainage of the wall cannot be achieved, add hydrostatic pressure assuming groundwater at 5 ft below top of wall.

We recommend the following additional requirements for the ground anchor (tieback):

- The proposed first row of ground anchor (tieback) should be installed at least 6.5 ft below the top of the wall and they should be installed at an angle of 15 degrees below the horizontal.
- The unbonded length of the tieback anchors should be a minimum of 40 ft long.

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- The bonded length of the ground anchor (tieback) should be left up to the contractor. The contractor is responsible for providing tieback anchors that satisfy the contract specifications.
- Pile spacing should be limited to no more than 8 ft.
- The ultimate and allowable vertical compression capacities of piles are specified in Table 8.
- Use 50 percent of the compression shaft resistance values mentioned above to calculate the ultimate tension (uplift) resistance of the pile.

VII.2. RETAINING WALL FOR MBGR TRANSITION TO TYPE 742 BARRIER

A wall is needed at each end of the tieback wall to retain the fill required to provide MBGR transition to Type 742 barrier proposed for this project. We concurred with Structure Design to use a modified Type 736 S/SV barrier on CIDH pile called concrete barrier Type 736 (Mod). The alignments of these walls are outside of the slide zone and the detail for Case 2 shown in 2010 revised Standard Plan can be used. For the pile spacing (S) and embedment (L) refer to plan sheet B15-8 using backfill friction angle of 30 degrees, slope of 2H:1V in front of the wall, and the fill thickness (He) equal to 4 ft behind the barrier. Please consult with us to determine the required pile embedment if the slope is steeper than 2H:1V. The backfill material shall conform to Standard Specifications Chapter 19 section for retaining walls.

VIII. DRAINAGE

We recommend to install horizontal drains of minimum 1.5 inches in diameters at spacing of 30 ft at about 1 ft above the FG of the lowest bench before placing the fill in front of wall. The horizontal drains shall be about 30 ft in length and installed at about 5 degree above the horizontal plane. The last 5 ft of the horizontal drain behind the wall shall be solid. All horizontal drains shall be discharged in a solid pipe of minimum 6 inch in diameter which installed on the wall. This pipe shall be outlet in a down drain pipe at every 400 ft length.

Please consult with District 4 Hydraulics Branch For discharge collection system of the horizontal drains, culvert repair and, details on accommodating the culvert extension through the proposed wall, placing rock dissipator at the culvert outlet, surface drainage and collection system.

IX. SHOTCRETE FACING VERSUS WALER BEAM OPTION

Soldier beams ground anchor (tieback) walls traditionally are designed with use of wood lagging and stiff walers to bridge the adjacent beams and transfer all the lateral earth pressure to the beam and tiebacks. These systems performed very successfully over many years due to its

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rigidity, load transfer mechanism between the shallower and deeper than the slide depths that assumed in the design, and its drainage ability. The above recommendations are for the soldier beam ground anchor (tieback) wall with wood lagging and waler beams system. If the use of shotcrete is proposed instead of the wood lagging and waler, we recommend the following:

- 1) Use three rows of ground anchor (tieback), each with design load of 13.2 Kips/ft and maximum test load of 17.2 kips/ft with the same unbounded length of 40 ft. This is to provide more uniform fixity between the shotcrete and the beams.
- 2) Use structural shotcrete with a minimum thickness of 13.5 inches to provide same rigidity as that of the soldier beam ground anchor (tieback) with wood lagging and waler beams wall system.
- 3) The structural shotcrete shall remained intact under the maximum earth pressure calculated for static pressure outline in Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.7 Figure 5.5.5.7.1-1b and that increased by a factor of 1.3 for the seismic loading conditions. In addition, the structural integrity of the entire beam and shotcrete system and shotcrete facing panels need to be checked by the designer for all of the excavation stages.
- 4) Both weep holes above the finished bench and the horizontal drains at a minimum spacing of 30 ft near the bottom the maxim excavation depth are required. Please refer to Drainage section in the above for collecting and discharging the horizontal drains.

X. CORROSION

Samples for corrosion testing were collected from the soil borings during the subsurface investigation. Four samples were collected from two borings for corrosion testing. Test results, shown in Table 9, have indicated that the site is not corrosive.

Table 9. Corrosion Results

Boring	Depth	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
RW-12-001	10	1404	8.4		
RW-12-001	53	5	9.18	1	122
RW-12-002	15	1554	8.52		
RW-12-002	62	4	8.89	46	32

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The pH and Resistivity test results indicate that the site is non-corrosive (per Caltrans Corrosion guide dated September 2003). Use Standard corrosion protection measures for this project. Corrosion test results are attached.

XI. CONSTRUCTION CONSIDERATIONS AND REQUIREMENTS

The following construction considerations and requirements should be included in the design and construction specifications for the proposed wall:

- During drilling of CIDH pile, ground anchor (tieback) holes, and horizontal drain holes, the contractor may encounter bedrock of varying hardness due to presence of interbedded sandstone and mudstone at the site. The boring logs indicate that the site is underlain primarily by soft mudstone bedrock; however, based on the geologic map for the area and local rock exposures, hard to very hard sandstone layers of varying thickness may be encountered during drilling. The contractor is required to use drilling and cutting bits suitable for soft mudstone and hard to very hard sandstone bedrock. Additionally the contractor shall be prepared to drill through the previously installed reinforced HDPE geogrids.
- Due to the presence of high groundwater, temporary casing of the drill holes and dewatering may be required.
- Variation in the drilling advance rate shall be expected due to variable soil and rock conditions both in term of type, hardness and depth encountered.
- During drilling operation for the proposed soldier beam piles, we believe that some caving of the drilled holes will likely occur. This also dictates use of casing combined with dewatering.
- Water that has infiltrated the hole shall be removed before placing concrete therein. Fluvial or drainage water shall not be permitted to enter the hole.
- Backfill of a portion of the wall might be required. For this portion structure fill shall used and be compacted in lifts not thicker than one foot to 95% relative compaction before placement of tieback. Light compaction equipment shall be used near the wall face.
- Installation of the CIDH piles should be performed in accordance with Section 49-4 of the 2010 Caltrans Standard Specifications.
- The drilling and concrete placement for CIDH pile construction shall be staggered. No open holes shall be adjacent.

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- All excavation shall comply to Cal-Osha requirements.

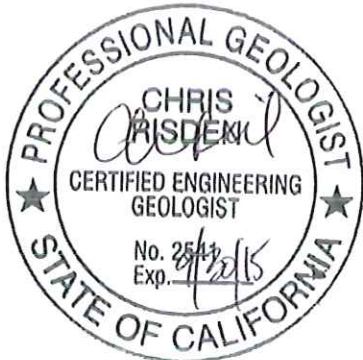
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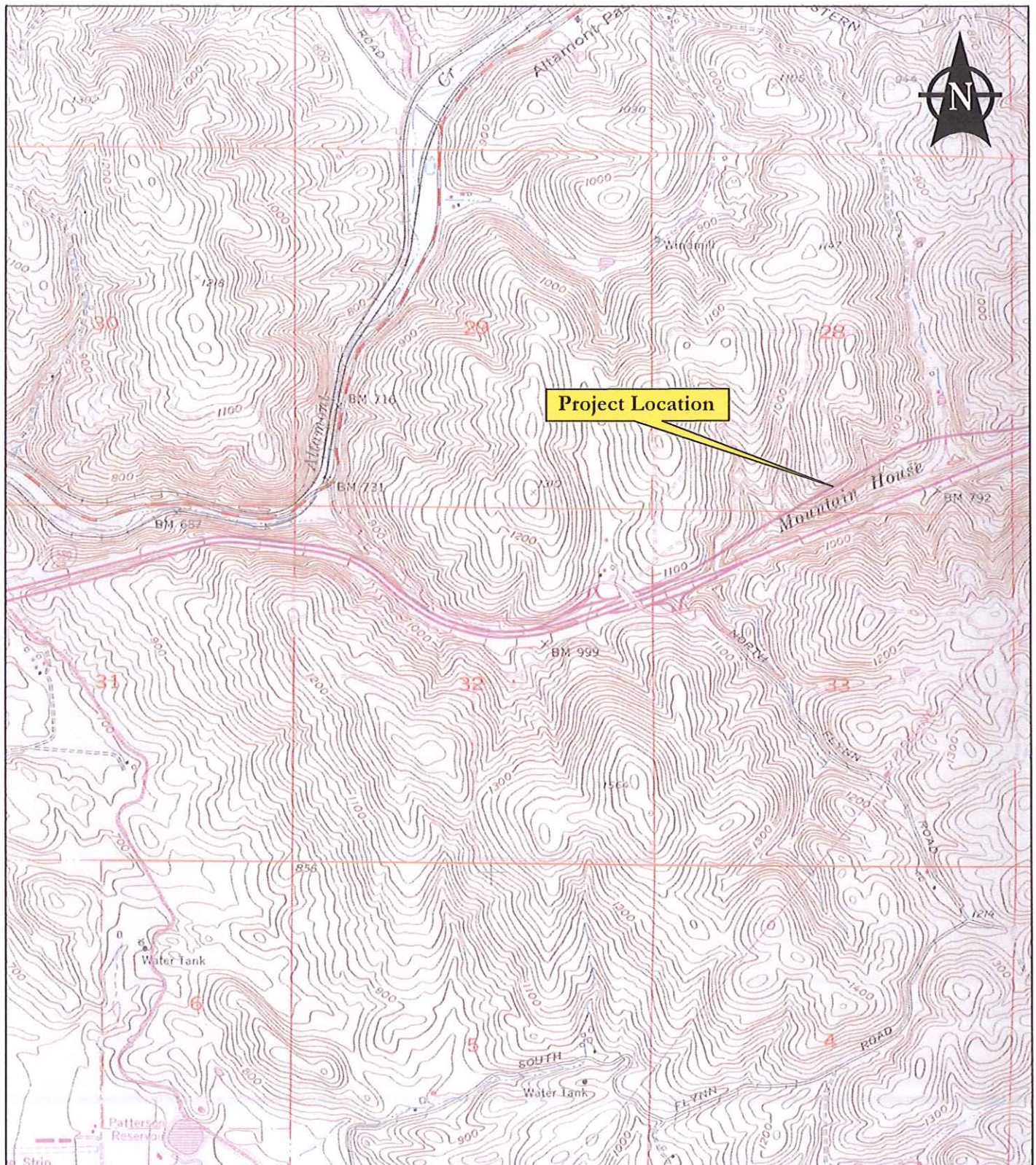
Should you have any questions, please call me at (510) 286-4821.

Attachments

c: TPokrywka, MMomenzadeh, CRIsden, IYalan, RKarpowitz, JMoore, EOrtega, Henry_Seto,
Ruth Fernandes, Melanie_Brent, RE_Pending_File@dot.ca.gov, Tom_Whitman Tinu_Mishra

Eduardo Ortega/mm/FR ALA 580 PM 5 WB Revised.docx





Base: USGS Topographic Map, 7.5' Altamont Quadrangle, 1981
 Scale: 1 inch = 2,000 feet



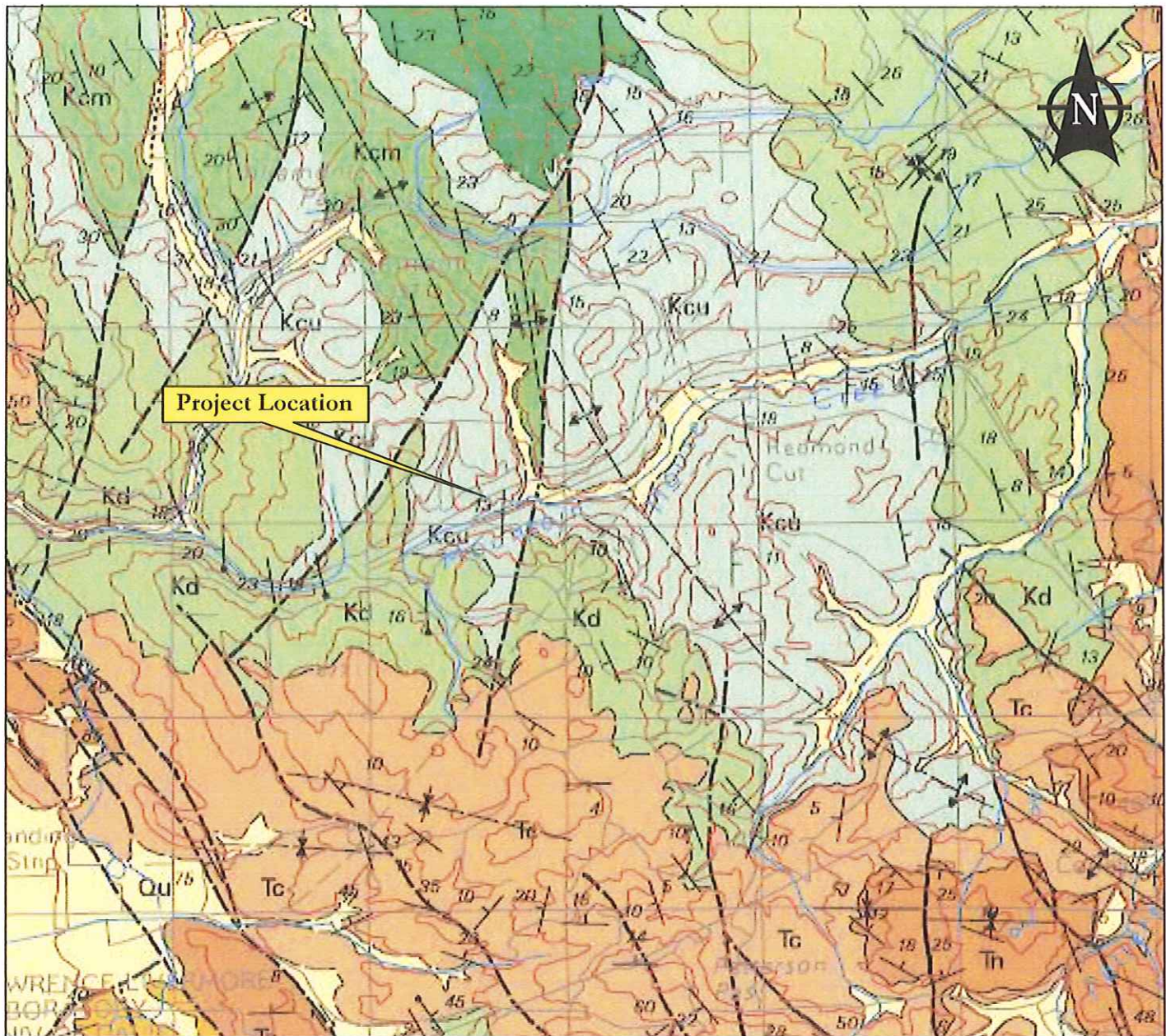
**STORM DAMAGE
 HIGHWAY 580 PM 5.0
 ALAMEDA COUNTY, CALIFORNIA**

LOCATION MAP

EFIS#: 0400020869

MAY 2014

FIGURE 1



LEGEND



Neroly Sandstone



Unnamed Shale and Siltstone



Cierbo Sandstone



Unnamed Wacke



Unnamed Sandstone

Base: Preliminary Geologic Map Emphasizing Bedrock Formations
in Alameda County, California (Graymer and others, 1996)
Scale: 1:75,000



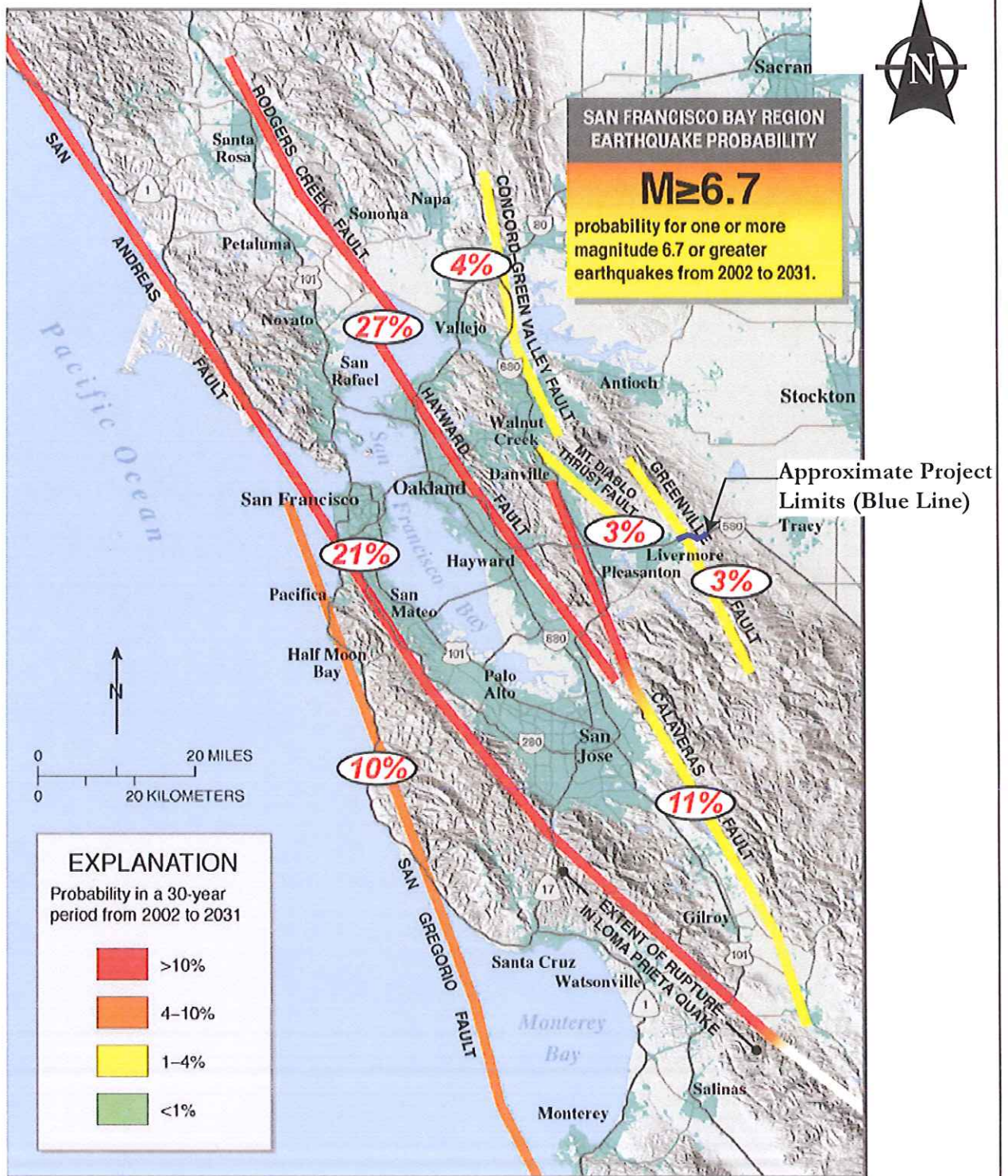
STORM DAMAGE
HIGHWAY 580 PM 5.0
ALAMEDA COUNTY, CALIFORNIA

GEOLOGIC MAP

EFIS#: 0400020869

MAY 2014

FIGURE 2



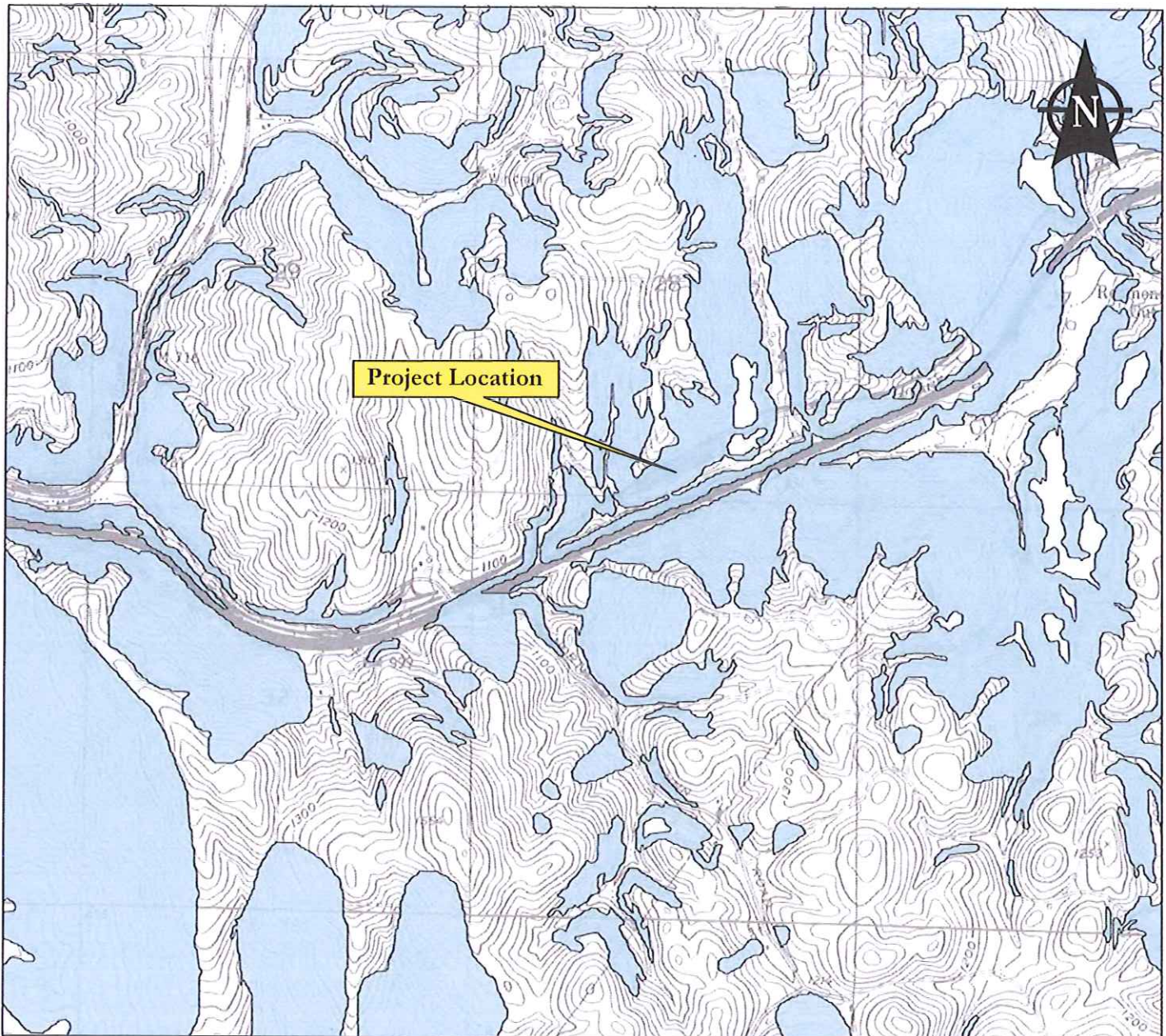
**STORM DAMAGE
HIGHWAY 580 PM 5.0
ALAMEDA COUNTY, CALIFORNIA**

**SF BAY REGION
EARTHQUAKE
PROBABILITY**

EFIS#: 0400020869

MAY 2014

FIGURE 3



LEGEND



Areas where historic occurrence of liquefaction, or local, geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Base: State of California Preliminary Seismic Hazard Zone Map for the Altamont 7.5 Minute Quadrangle, CGS 2008.

Scale: 1 inch = 2,000 feet



STORM DAMAGE
HIGHWAY 580 PM 5.0
ALAMEDA COUNTY, CALIFORNIA

STATE SEISMIC
HAZARD MAP

EFIS#: 0400020869

MAY 2014

FIGURE 4

E.ORTEGA
Geotechnical Design - West
04-ALA-580 PM 5.0 EB
5/14/2014

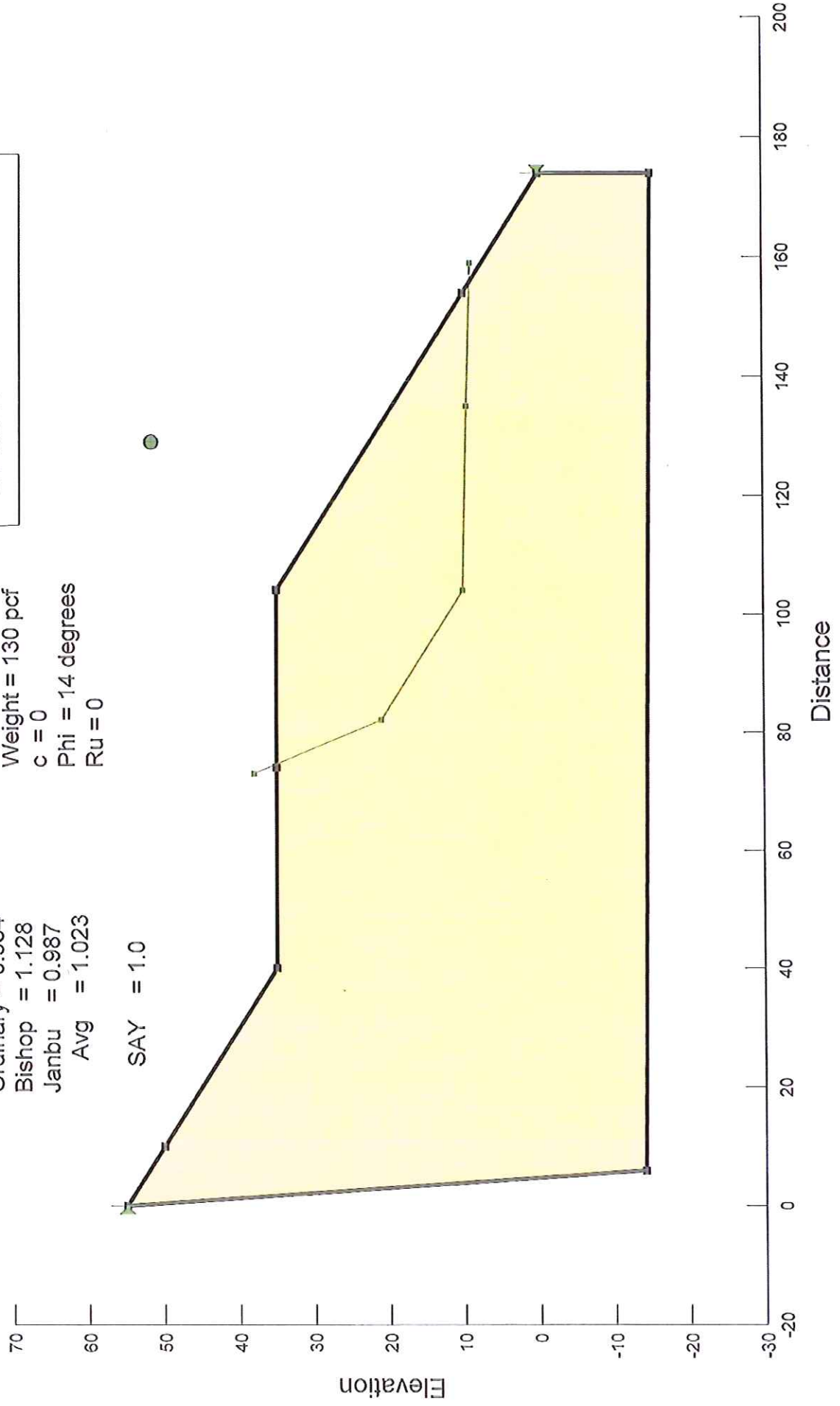
SAFETY FACTORS

Ordinary = 0.954
Bishop = 1.128
Janbu = 0.987
Avg = 1.023

SAY = 1.0

SOIL

Weight = 130 pcf
 $c = 0$
 $\Phi = 14$ degrees
 $R_u = 0$



E.ORTEGA
 Geotechnical Design - West
 04-ALA-580 PM 5.0 EB
 5/14/2014

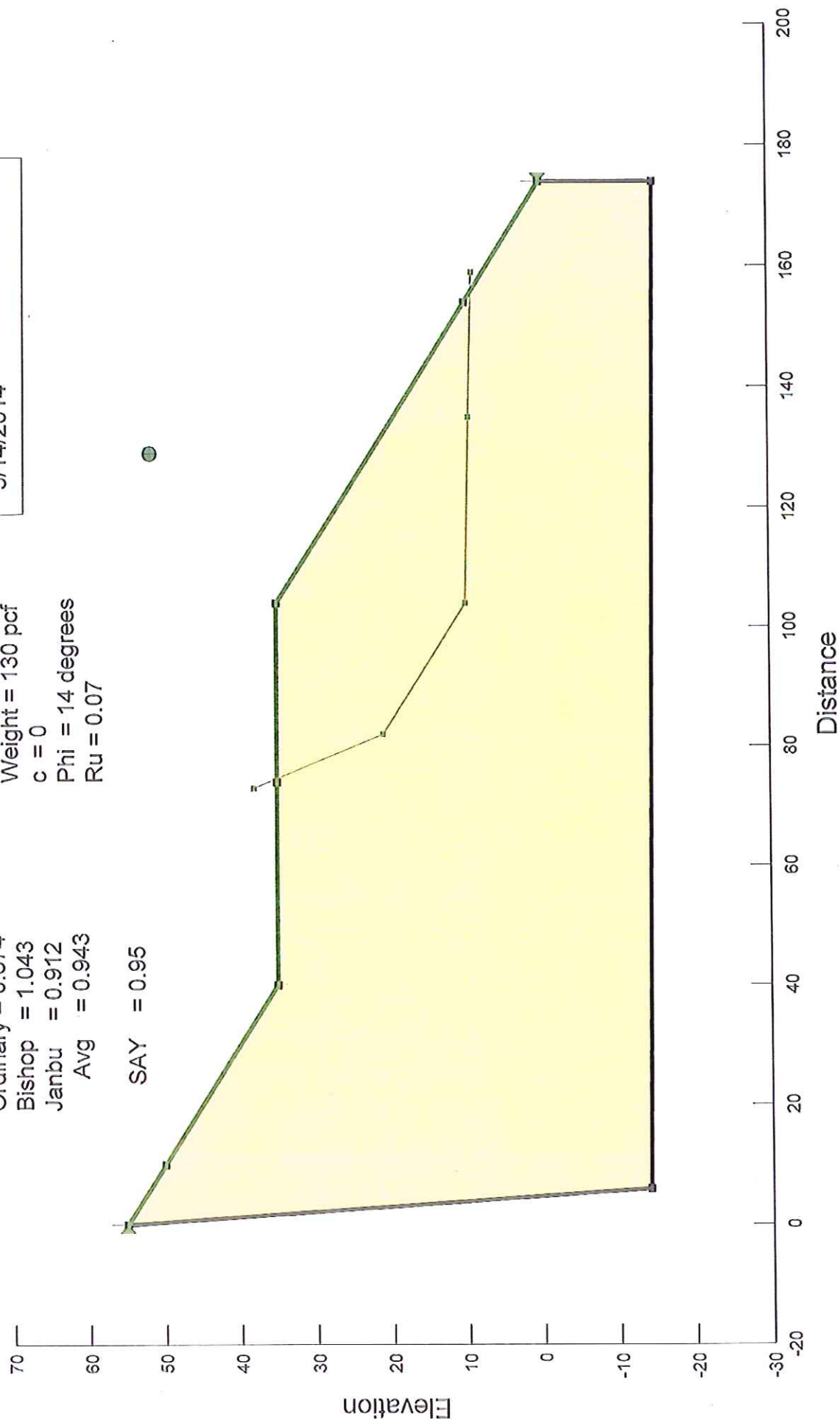
SAFETY FACTORS

Ordinary = 0.874
 Bishop = 1.043
 Janbu = 0.912
 Avg = 0.943

SAY = 0.95

SOIL

Weight = 130 pcf
 $c = 0$
 $\Phi = 14$ degrees
 $R_u = 0.07$



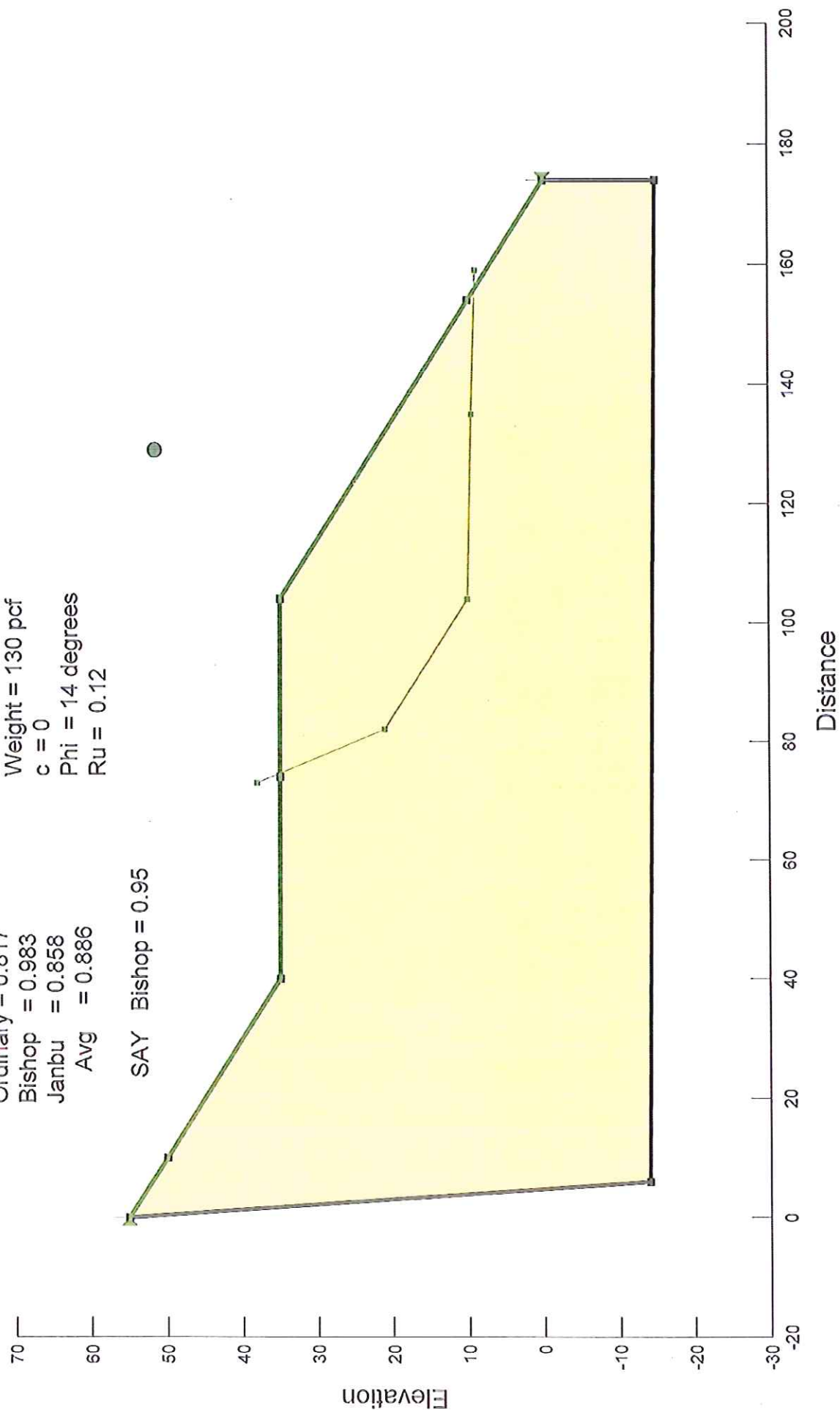
SAFETY FACTORS

Ordinary = 0.817
Bishop = 0.983
Janbu = 0.858
Avg = 0.886

SAY Bishop = 0.95

SOIL

Weight = 130 pcf
 $c = 0$
 $\Phi = 14$ degrees
 $R_u = 0.12$



SAFETY FACTORS

Ordinary = 1.307
 Bishop = 1.354
 Janbu = 1,318

Avg = 1.326 > 1.3 OK

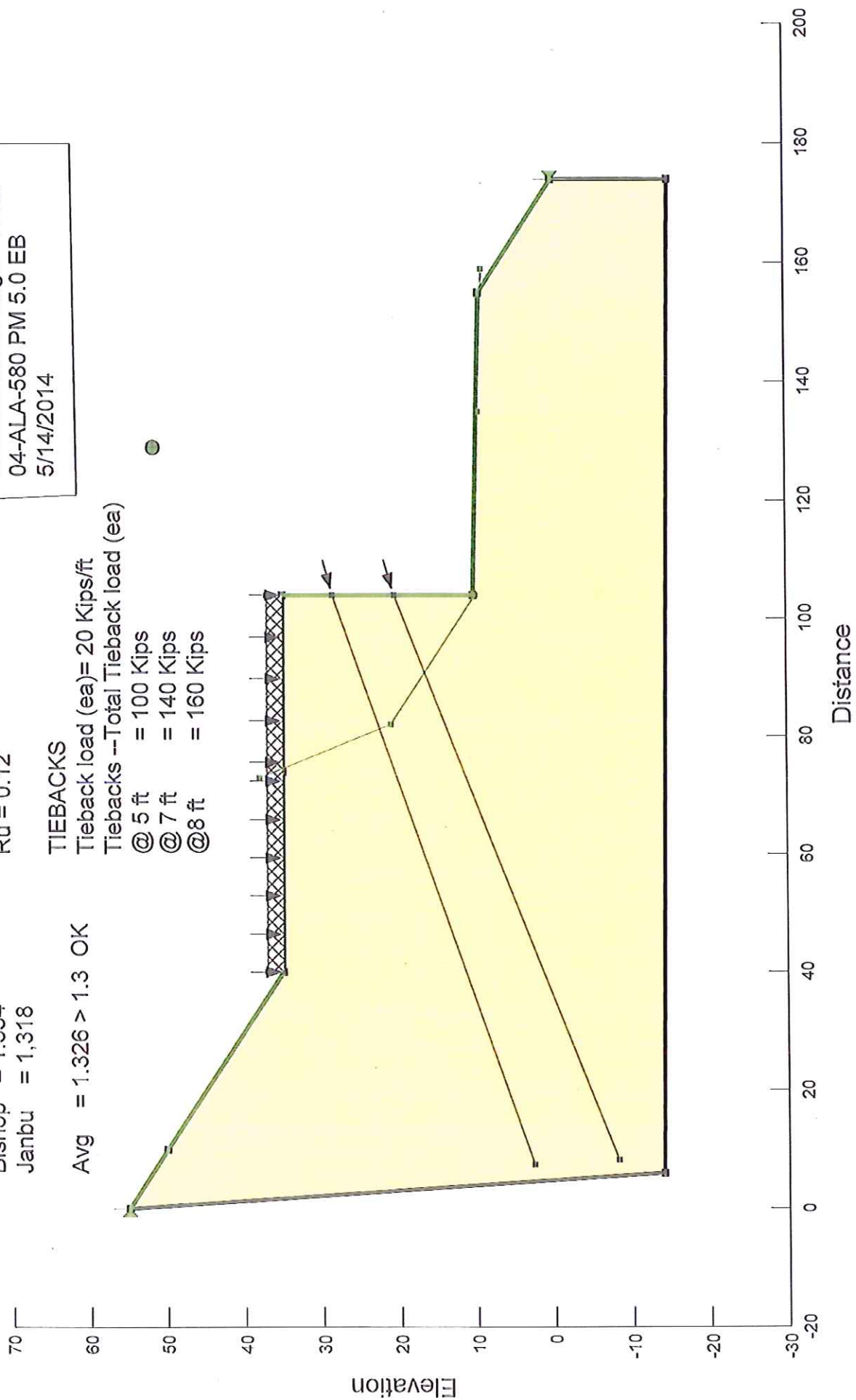
SOIL

Weight = 130 pcf
 c = 0
 Phi = 14 degrees
 Ru = 0.12

TIEBACKS

Tieback load (ea) = 20 Kips/ft
 Tiebacks -- Total Tieback load (ea)
 @ 5 ft = 100 Kips
 @ 7 ft = 140 Kips
 @ 8 ft = 160 Kips

E. ORTEGA
 Geotechnical Design - West
 04-ALA-580 PM 5.0 EB
 5/14/2014



SEISMIC 0.408g (Greenville-ID 131)

$$c = 0.408g/3 = 0.136 g$$

SOIL

Weight = 130 pcf

$c = 0$

$\Phi = 14$ degrees

$R_u = 0.12$

SAFETY FACTORS

Ordinary = 1.417

Bishop = 1.510

Janbu = 1.343

Avg = 1.423 > 1.1 OK

TIEBACKS

Tieback load (ea) = 22 x 1.3 = 26 Kips/ft

Tiebacks -- Total Tieback load (ea)

@ 5 ft = 130 Kips

@ 7 ft = 182 Kips

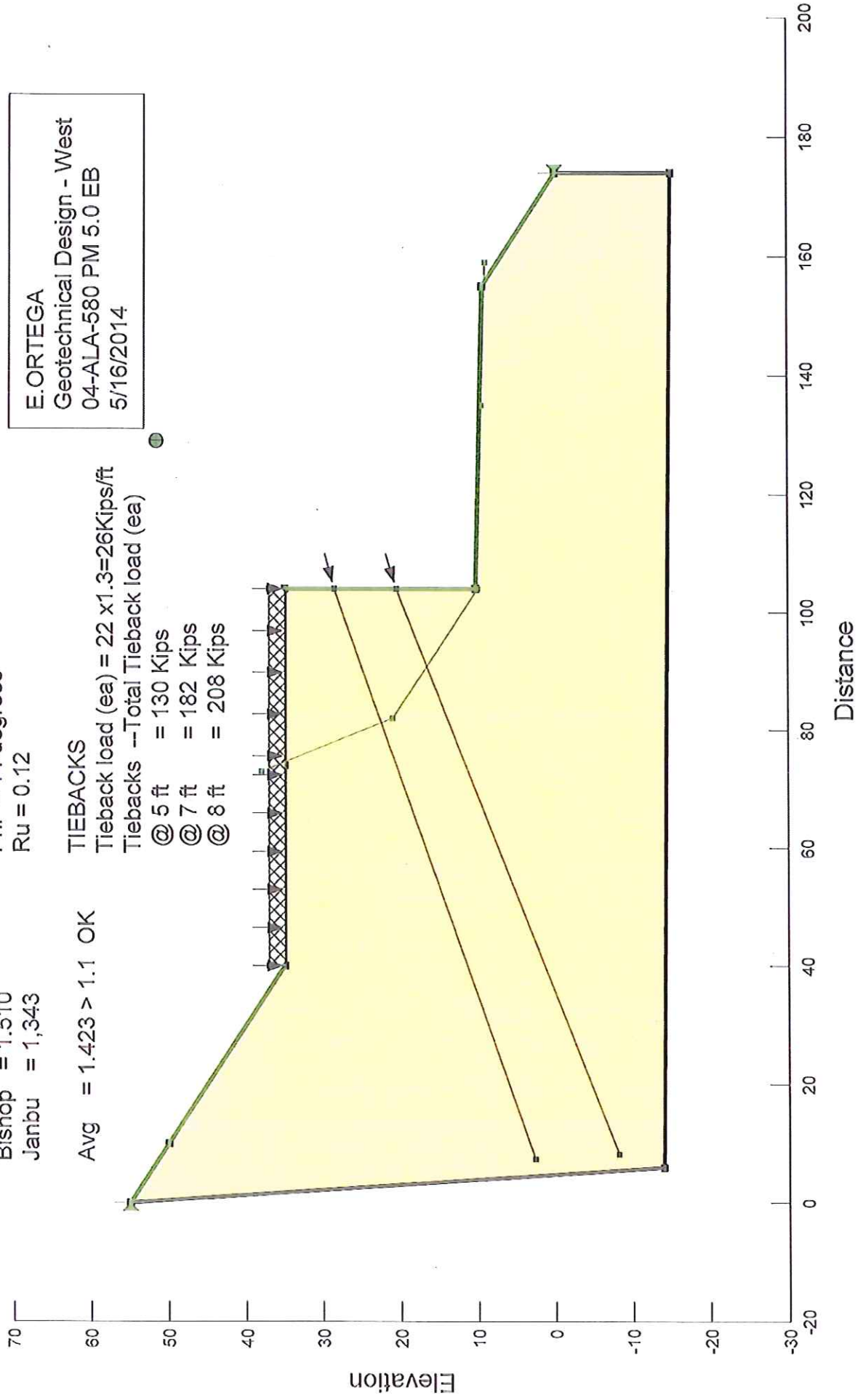
@ 8 ft = 208 Kips

E. ORTEGA

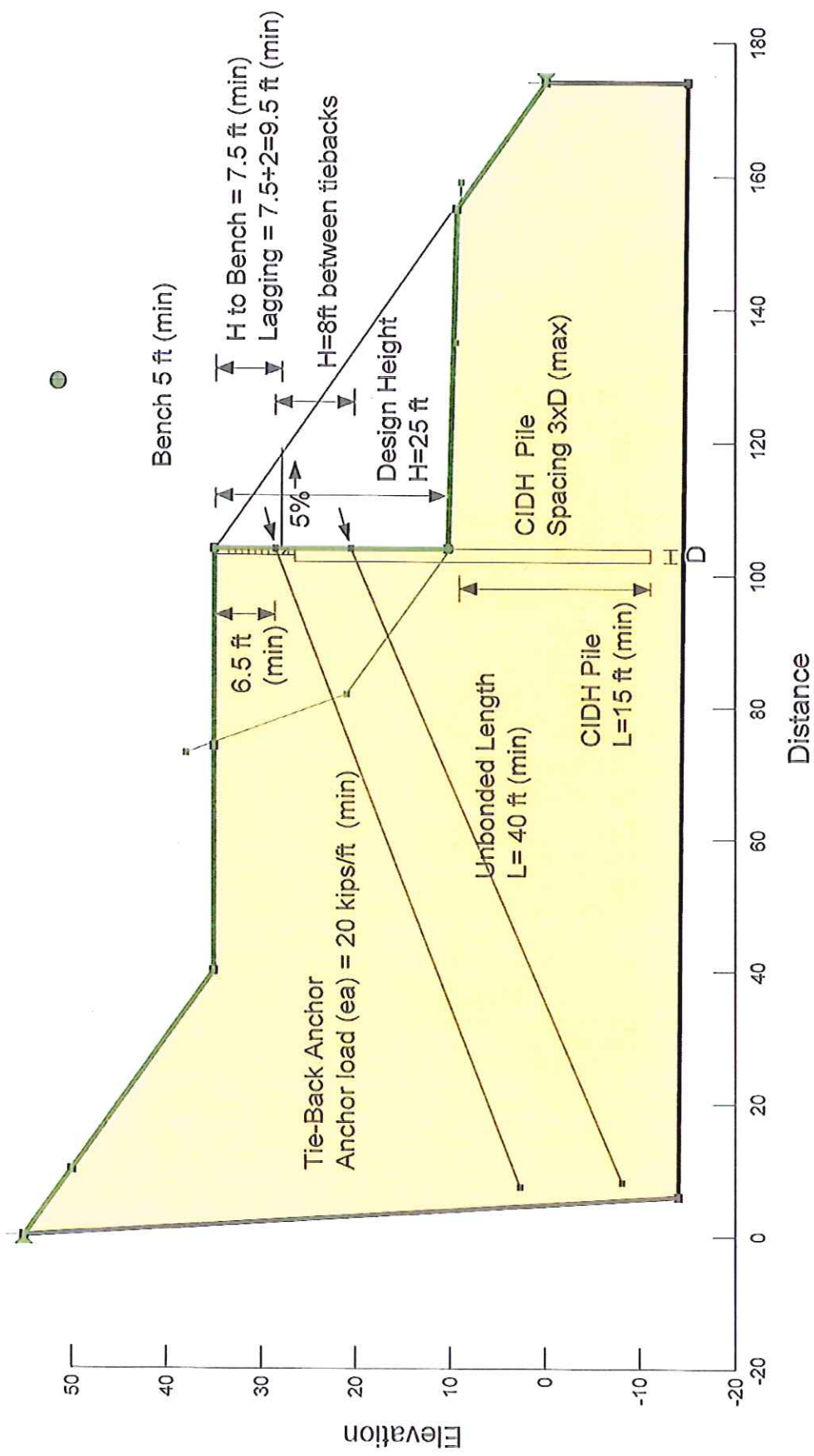
Geotechnical Design - West

04-ALA-580 PM 5.0 EB

5/16/2014



Office of Geotechnical Design - West Geotechnical Services Division of Engineering Services	STORM DAMAGE REPAIR TYPICAL X SECTION	04-ALA-580 PM 5.0 EB EA 1SS030 EFIS#0400020869-0 5/16/2014
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5.5.5.7 Lateral Earth Pressures for Anchored Walls

For anchored walls restrained by tie rods and structural anchors, the lateral earth pressure acting on the wall may be determined in accordance with Article 5.5.5.6.

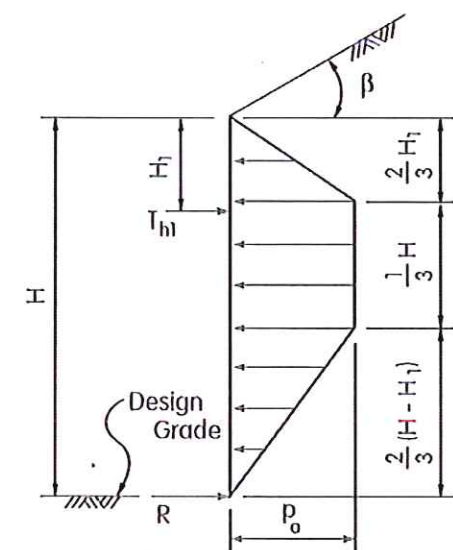
For anchored walls constructed from the top down and restrained by ground anchors (tieback anchors), the lateral earth pressure acting on the wall height, H , may be determined in accordance with Articles 5.5.5.7.1 and 5.5.5.7.2.

For anchored walls constructed from the bottom up and restrained by a single level of ground anchors located not more than one third of the wall height, H , above the bottom of the wall, the total lateral earth pressure, P_{Total} , acting on the wall height, H , may be determined in accordance with Article 5.5.5.7.1 with distribution assumed to be linearly proportional to depth and a maximum pressure equal to, $\frac{2P_{Total}}{H}$. For anchored walls constructed from the bottom up and restrained by multiple levels of ground anchors, the lateral earth pressure acting on the wall height, H , may be determined in accordance with Article 5.5.5.7.1.

In developing the lateral earth pressure for design of an anchored wall, consideration shall be given to wall displacements that may affect adjacent structures and/or underground utilities.

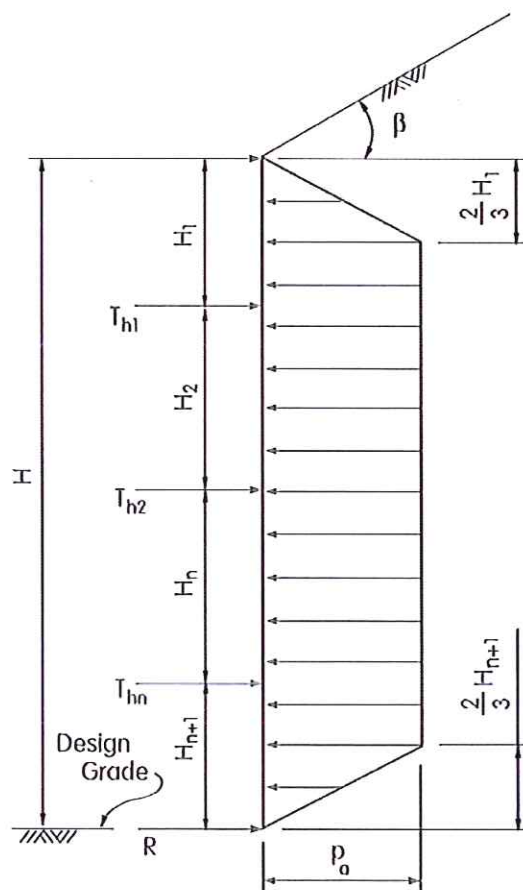
C5.5.5.7

In the development of lateral earth pressures, the method and sequence of wall construction, the rigidity of the wall/anchor system, the physical characteristics and stability of the ground mass to be supported/retained, allowable wall deflections, anchor spacing and prestress and the potential for anchor yield should be considered.



Note: $H_1 \leq \frac{2}{3} H$

a) Wall with a single level of anchors



b) Wall with multiple levels of anchors

Figure 5.5.5.7.1-1 Lateral Earth Pressure Distributions for Anchored Walls Constructed from the Top Down in Cohesionless Soils

Looking east on WB I-580, Postmile 5.0 – pavement cracking in the shoulder adjacent to the Number 1 lane.



Looking west on WB I-580, Postmile 5.0 – pavement cracking in the shoulder adjacent to the Number 1 lane.



The current project consists of constructing a retaining wall to restrain the slope.

Moore, J C@DOT

From: Conway, Wendy@DOT
Sent: Thursday, February 27, 2014 11:56 AM
To: Moore, J C@DOT
Subject: north flynn update
Attachments: North Flynn GW.xlsm; NFLYNN RW003 Feb2014.pdf; NFLYNN RW002 Feb2014.pdf

wendy conway

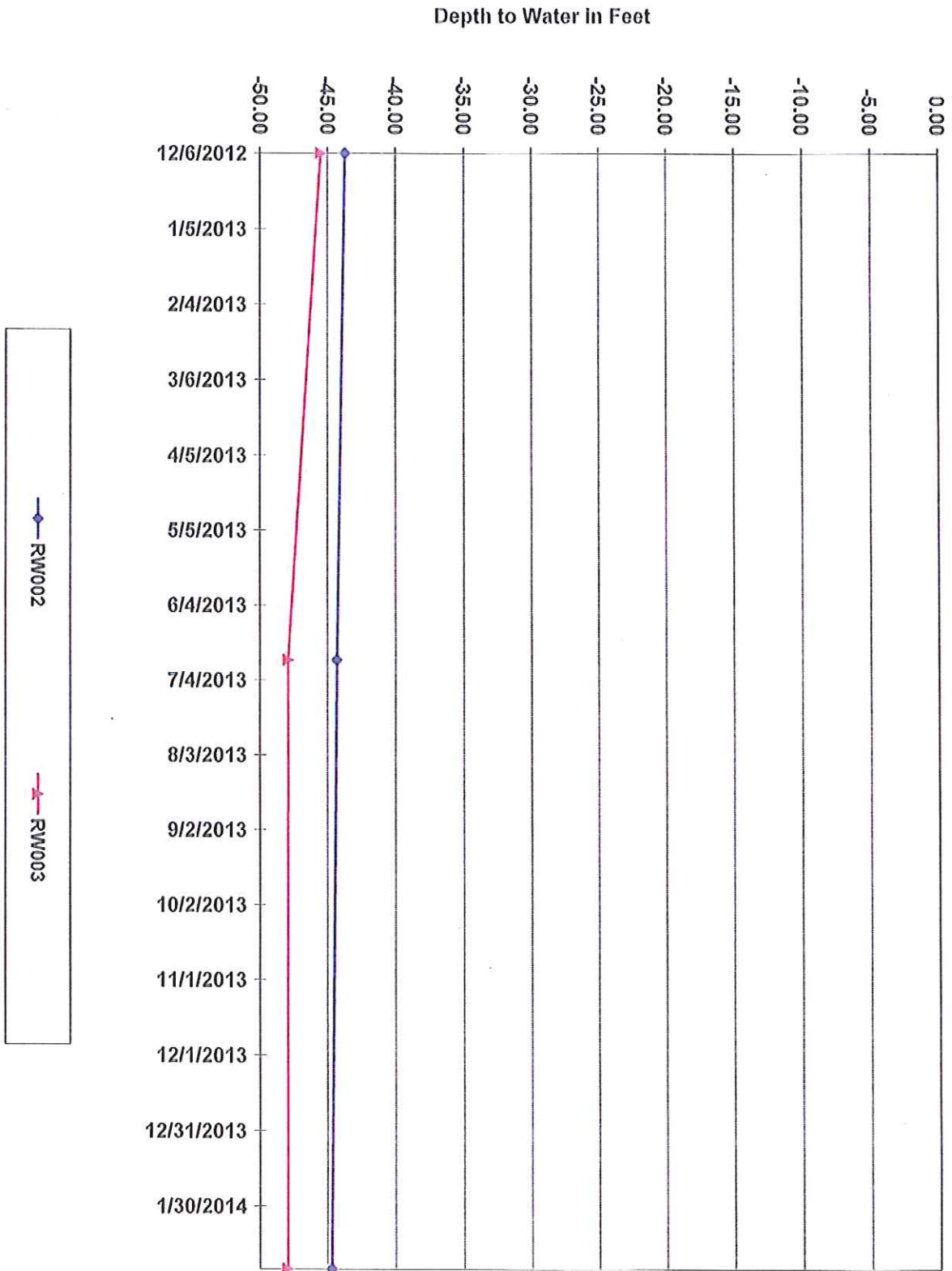
mrea/geotech design west

d4 - oakland

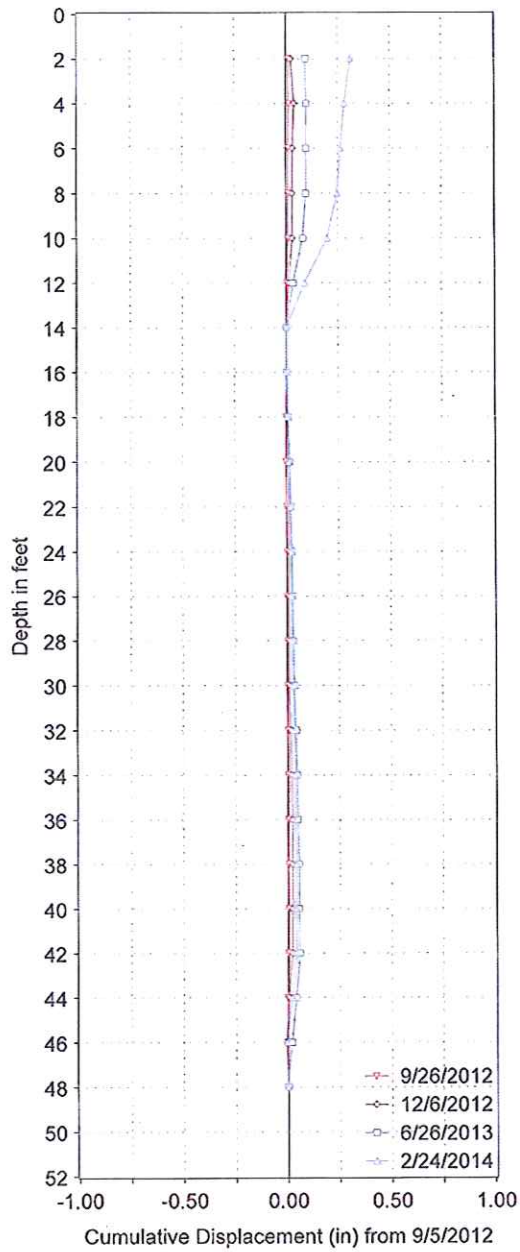
wconway@dot.ca.gov

cell # 510-292-5177

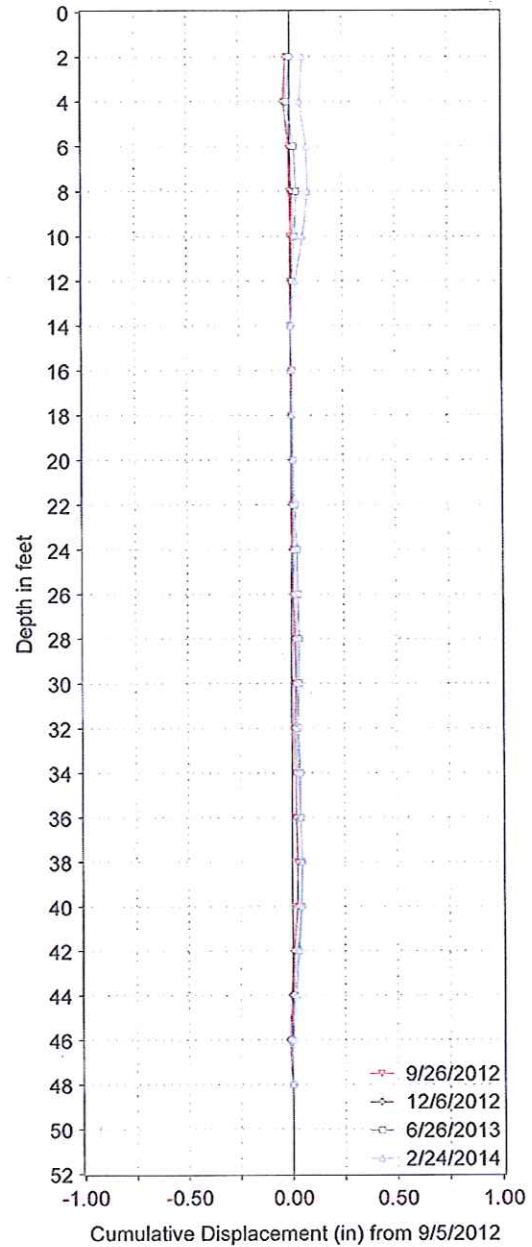
NORTH FLYNN GROUNDWATER DATA
Readings from 12/2012 to present



NFLYNN RW002, A-Axis



NFLYNN RW002, B-Axis

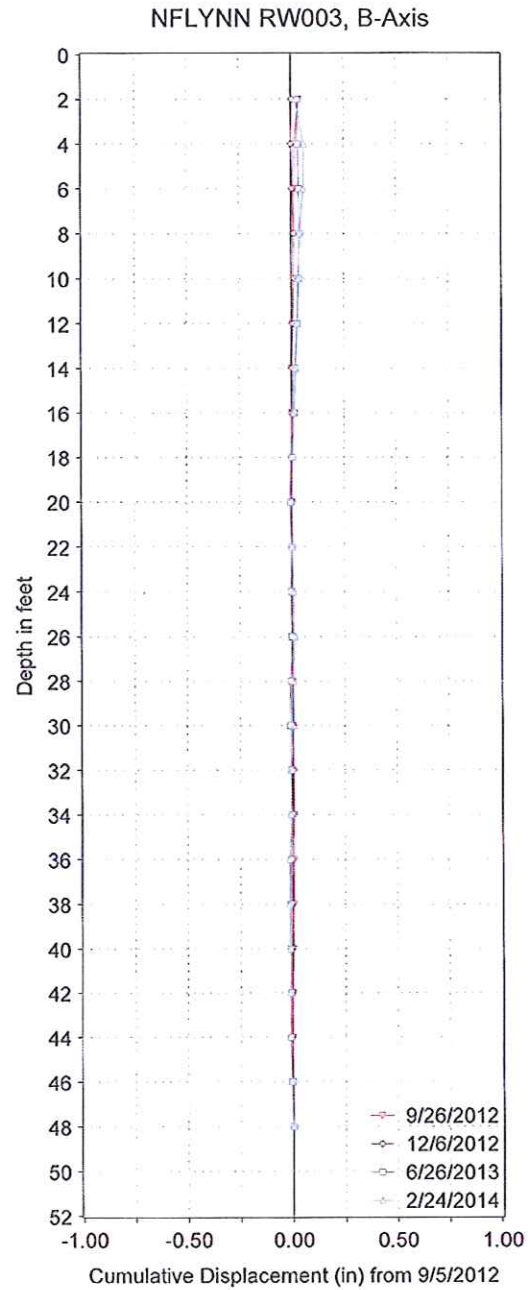
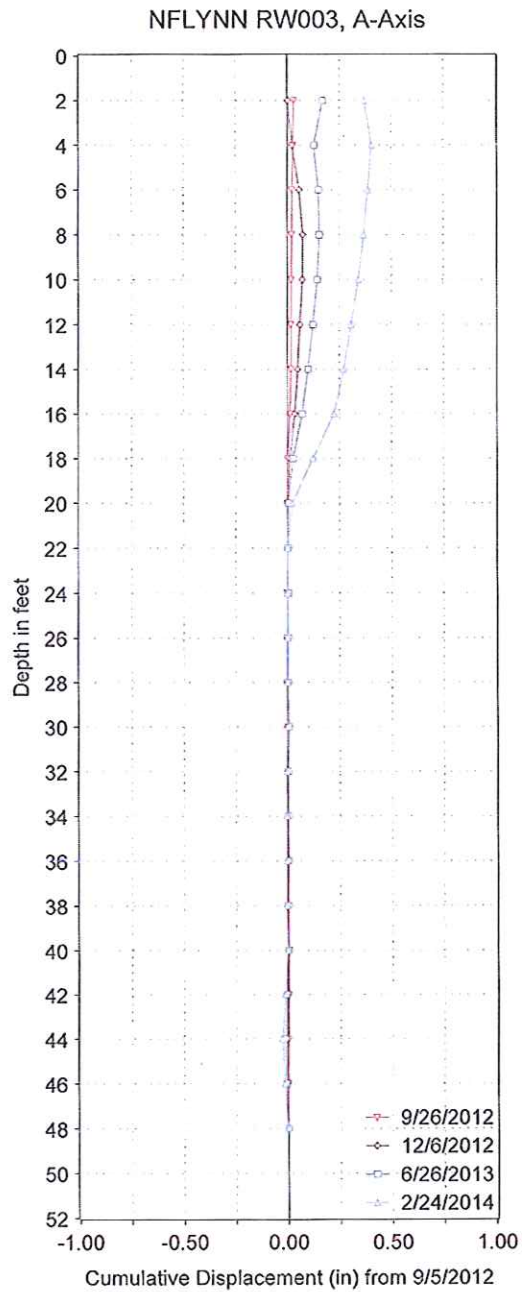


NORTH FLYNN WESTBOUND

04-ALA-580-PM-5.0

Initial Reading taken on 9/5/2012

Bias shift applied



NORTH FLYNN WESTBOUND

04-ALA-580-PM 5.0

Initial Reading Taken on 9/5/2012

Bias shift applied

Results sent to: JOHN MOORE

Division of Engineering Services
Materials Engineering and Testing Services
Corrosion and Structural Concrete Field Investigation Branch
Report Date: 8/20/2012
Reported by Michael Mirkovic

TEST SUMMARY REPORT - SOIL

EA 04-1SS03X

EFIS: 0400020869

Dist/Co/Rte/PM: 04 / ALA /580/ / 5 PM

DISCOUNT W. 047 R.R. 10007 131 MI.

CORROSION LAB #	TL101 #	BORE #	DEPTH (FT)		MINIMUM RESISTIVITY ¹ (ohm-cm)	pH ¹	CHLORIDE CONTENT ² (ppm)	SULFATE CONTENT ³ (ppm)	IS SAMPLE CORROSIVE?
			START	END					
SOIL SAMPLE FROM: WB LEFT SIDE SHOULDER									
CR20120335	C232332	RW-12-003	20	20	1183	8.55			NO
CR20120336	C232333	RW-12-002	35	40	1778	8.39			NO
CR20120337	C232334	RW-12-004	40	45	2049	8.66			NO

This site is not corrosive to foundation elements(see note below for MSE wall backfill).

Note: For MSE wall structure backfill material, minimum resistivity must be 2000 ohm-cm or greater, pH must be between 5.5 and 10.0, chloride content must not be greater than 250 ppm, and sulfate content must not be greater than 500 ppm.

¹CTM 643, ²CTM 422, ³CTM 417

CR20120335 - CR20120337

8/20/2012



**DIVISION OF
ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY**
5900 Folsom Boulevard
Sacramento, CA 95819

Date: 9/12/2012

To: John Moore / GDW

From: Lilibeth C. Purta / (916) 227-5239

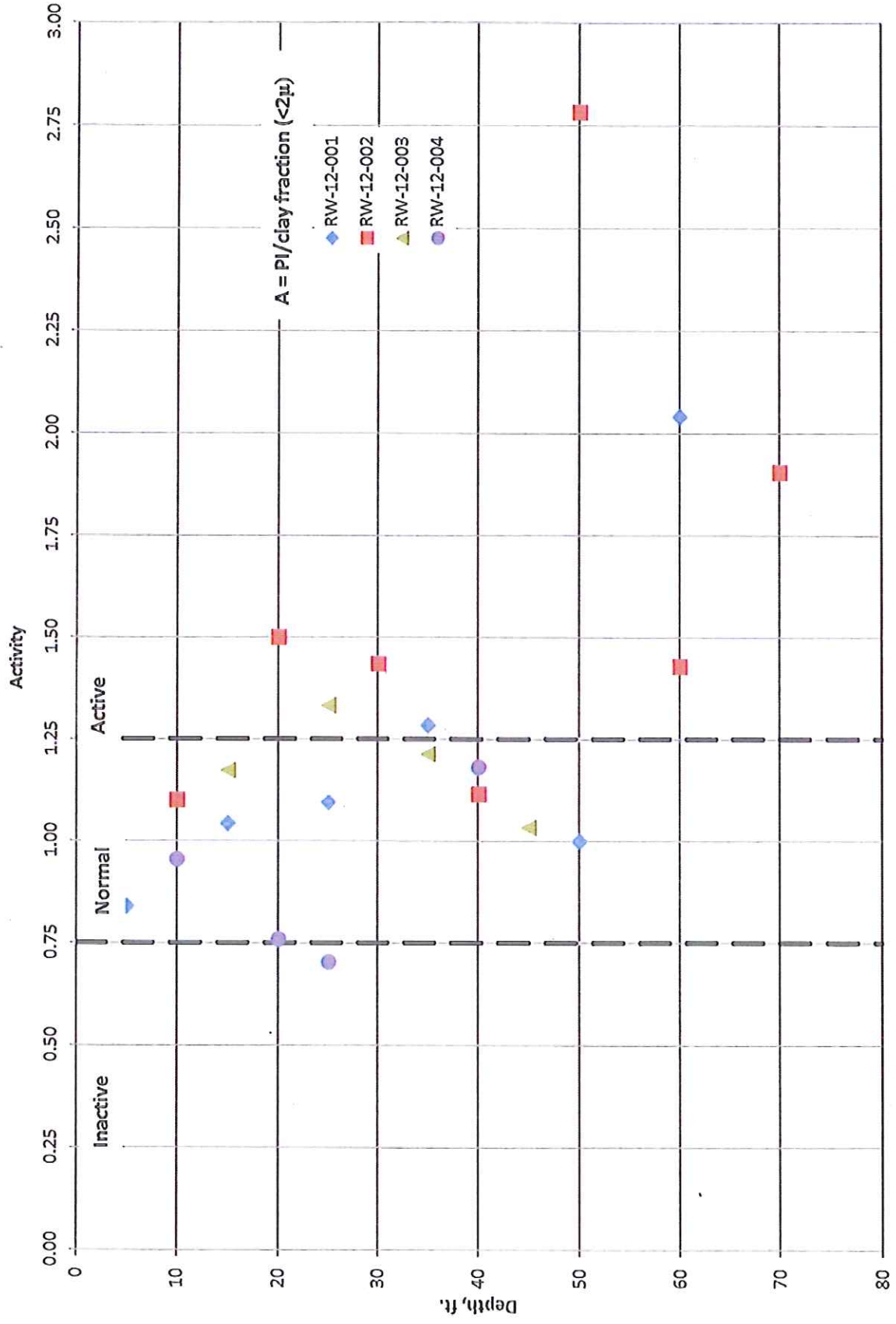
**RE: Laboratory Test Report --- EA: 04-2G8500
Project: 0412000008
GL 12-048**

Final test results.

**Note: All remaining test specimens will be disposed
of in 30 calendar days from the release date of the
final test results.**



Activity Vs. Depth





CLASSIFICATION TEST SUMMARY

SAMPLE ID	% FINER THAN																	ATTERBERG LIMITS			AS RECEIVED		Gs
	3"	2 1/2"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	5μ	1μ	LL	PI	Yd (pcf)	%m		
RW-12-001_05							100	96	92	91	90	89	88	87	86	44	25	43	21			23.4	
RW-12-001_10																							
RW-12-001_15							100	97	84	83	83	82	81	80	78	46	23	48	24			23.4	
RW-12-001_20																							
RW-12-001_25							100	95	81	80	79	79	77	75	74	45	21	47	23			22.6	
RW-12-001_30																							
RW-12-001_35																							
RW-12-001_40																							
RW-12-001_50						100	98	96	84	83	82	81	80	78	77	47	31	53	31			21.2	
RW-12-001_53																							
RW-12-001_60							100	99	93	91	88	86	83	81	79	49	24	72	49			21.9	
RW-12-001_70																							
RW-12-001_75																						19.4	
RW-12-001_85																						17.9	
RW-12-001_91																						19.4	
RW-12-002_05																							
RW-12-002_10							100	100	92	91	90	89	88	87	85	47	20	45	22			24.4	
RW-12-002_15																							
RW-12-002_20																							
RW-12-002_25																							
RW-12-002_30						100	98	94	82	80	79	78	77	75	73	45	23	54	33			24.6	
RW-12-002_35																							
RW-12-002_40																							
RW-12-002_45																							
RW-12-002_50						100	98	98	94	91	90	89	89	88	86	57	23	90	64			22.1	
RW-12-002_60										100	99	98	97	95	90	63	35	69	50			22.9	
RW-12-002_62																							
RW-12-002_70													100	99	96	51	21	65	40			20.5	



CLASSIFICATION TEST SUMMARY

[illegible]



**DIVISION OF
ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY**

5900 Folsom Boulevard
Sacramento, CA 95819

Date: 9/14/2012
To: John Moore / GDW
From: Lilibeth C. Purta / (916) 227-5239
RE: Laboratory Test Report -- EA: 04-2G8500
Project: 0412000008
GL 12-049

Final test results.

**Note: All remaining test specimens will be disposed
of in 30 calendar days from the release date of the
final test results.**





GL TRACKING NO : 12-049
Dist - EA: 04-2G8500
Report Date: September 14, 2012
Page: 1/1

[illegible]

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY

JOB LOCATION	04-ALA-580	PM 5.0	GL No.	12-056	DATE	8/14/2012
JOB NUMBER	04-1SS030	Westbound I-580, PM 5.0	Bridge No.		TEST BY	AZM
					CHECKED BY	LP 8/30/12

Note: No moistures recorded

*** The test specimen length/diameter ratio was not in compliance with the test method



**DIVISION OF
ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY**
5900 Folsom Boulevard
Sacramento, CA 95819

Date: 9/19/2012

To: John Moore / GDW

From: Lilibeth C. Purta / (916) 227-5239

**RE: Laboratory Test Report -- EA: 04-2G8500
Project: 0412000008
GL 12-051**

Final test results.

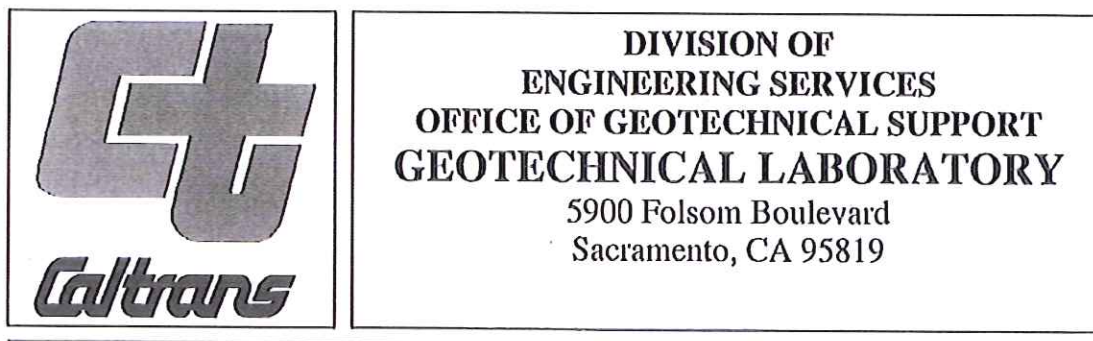
Note: All remaining test specimens will be disposed of in 30 calendar days from the release date of the final test results.





CLASSIFICATION TEST SUMMARY

[illegible]



Date: 9/19/2012

To: John Moore / GDW

From: Lilibeth C. Purta / (916) 227-5239

RE: Laboratory Test Report -- EA: 04-1SS030
Project: 0400020869
GL 12-052

Final test results.

Note: All remaining test specimens will be disposed of in 30 calendar days from the release date of the final test results.





GL TRACKING NO : 12-052
Dist - EA: 04-1SS030
Report Date: September 19, 2012
Page: 1/1

CLASSIFICATION TEST SUMMARY

[illegible]



**DIVISION OF
ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY**
5900 Folsom Boulevard
Sacramento, CA 95819

Date: 9/24/2012
To: John Moore / GDW
From: Lilibeth C. Purta / (916) 227-5239
RE: Laboratory Test Report -- EA: 04-1SS030
Project: 0400020869
GL 12-056

Final test results.

Note: All remaining test specimens will be disposed of in 30 calendar days from the release date of the final test results.





GL TRACKING NO : 12-056
Dist - EA: 04-1SS030
Report Date: September 24, 2012
Page: 1/1

CLASSIFICATION TEST SUMMARY

[illegible]